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Volume 5 | Technical Appendices

CFA25/26 | Castle Bromwich to Curzon Street

River modelling of the River Tame technical report (WR-004-019)

Water resources

November 2013

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Appendix WR-004-019

Environmental topic:	Water resources and flood risk assessment	WR
Appendix name:	Hydraulic modelling report	004
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1 Introduction

1.1 Structure of the water resources and flood risk assessment appendices

1.1.1 The water resources and flood risk assessment appendices comprise a number of parts. The first of these is a route-wide appendix (Volume 5: Appendix WR-001-000).

1.1.2 A number of specific appendices for each community forum area are also provided. For the Castle Bromwich and Bromford area (CFA25) and Washwood Heath to Curzon Street area (CFA26) these are:

- water resources assessments (Volume 5: Appendix WR-002-025 and WR-002-026);
- flood risk assessments (Volume 5: Appendix WR-003-025 and WR-003-026)
- a hydraulic modelling report for the River Tame (this Appendix);
- a hydraulic modelling report for the River Rea (Volume 5: Appendix WR-004-021); and
- a groundwater modelling report for the Bromford tunnel portals (Volume 5: Appendix WR-004-020).

1.1.3 Maps referred to throughout the water resources and flood risk assessment appendices are contained in the Volume 5 water resources map book.

1.2 Scope of this assessment

1.2.1 Hydraulic models were constructed to enable an assessment of a) the baseline "as-is" condition and b) with the Proposed Scheme included, to allow for review of impacts on flood risk. In order to assess the scheme the following has been undertaken:

- model a range of return periods from 50% AEP to 1% AEP plus climate change for the pre- and post-development situations to ascertain peak flood levels and flood extent;
- develop mitigation options for post-development; and
- inform land required for the scheme.

1.3 Location

1.3.1 This report focuses on CFA25 Castle Bromwich and Bromford and Washwood Heath to Curzon Street area (CFA26). The areas of consideration are shown in Figure 1 and Figure 2.

Figure 1: Castle Bromwich and Bromford

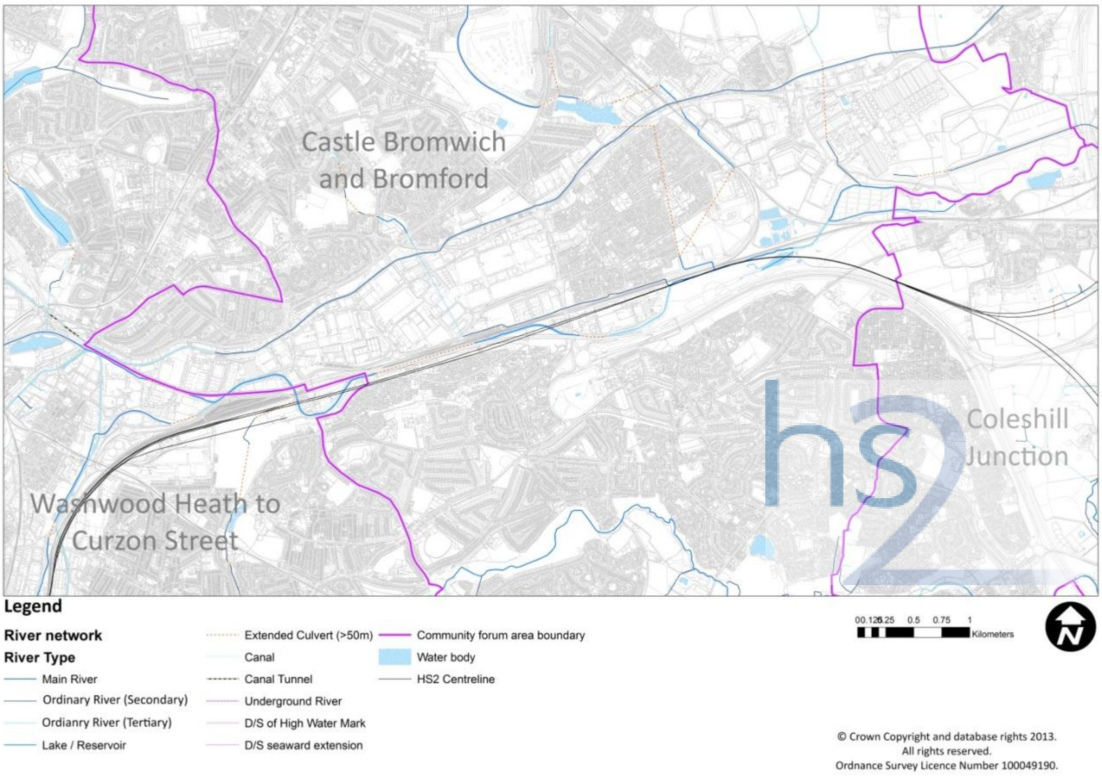
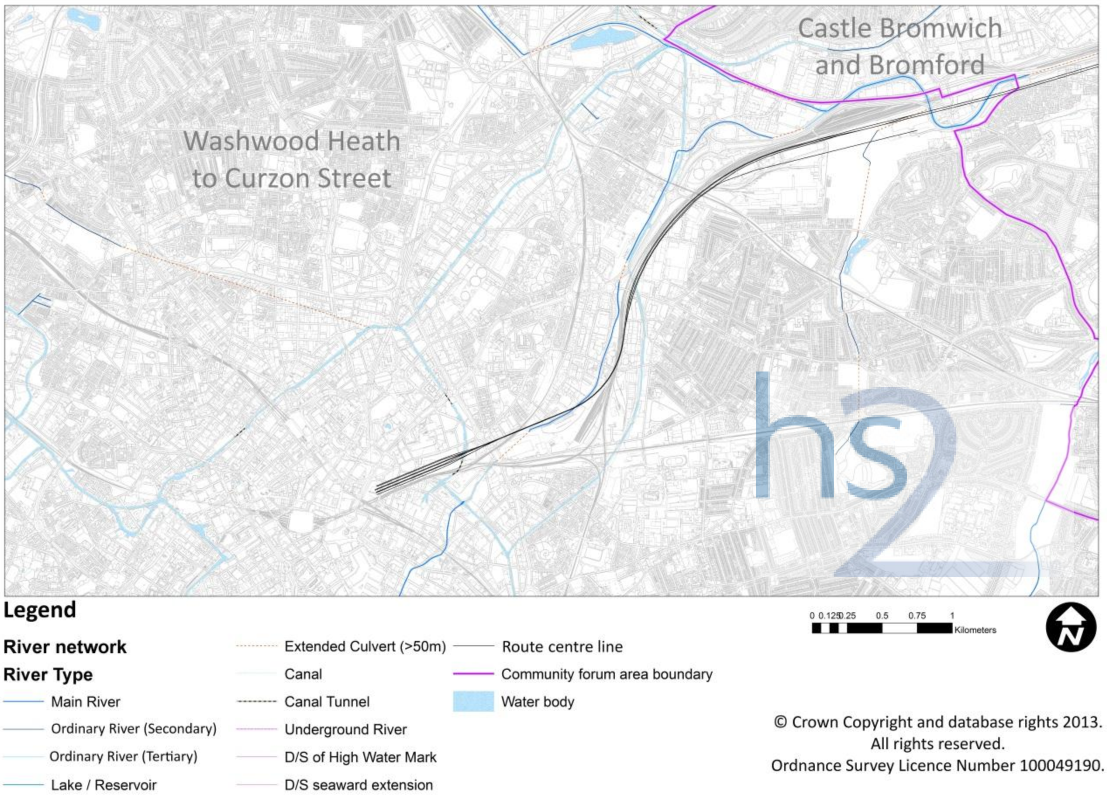


Figure 2: Washwood Heath to Curzon Street



2 Overview

2.1.1 The impact on flood risk due to the Proposed Scheme has been reviewed and options to mitigate increase in flood risk have been developed. The Proposed Scheme includes the main line, additional bridges and diversions of rivers and is detailed in the flood risk assessments, FRAs, (see Volume 5: Appendices WR-003-025 and WR-003-026).

2.1.2 This report focuses on the flood risk from the River Tame, which flows through Birmingham. It includes an overview of the available data, the current flood risk and the flood risk with the Proposed Scheme in place, including development of mitigation options.

2.2 Problem definition

2.2.1 The Proposed Scheme is a major infrastructure which will interact with the River Tame including:

- main line track on embankment and in cutting;
- a river diversion through Park Hall nature reserve;
- a viaduct crossing in Park Hall nature reserve; and
- construction of a depot at Washwood Heath

2.2.2 To understand the impact that the Proposed Scheme will have on flood risk, and to inform mitigation measures, it was apparent that a hydraulic model would be required.

2.3 Sources of data

2.3.1 A number of data sources were referenced as part of these works and are outlined as follows:

2.3.2 Environment Agency Midlands Region River Tame Strategic Flood Risk Mapping (SFRM) Study (Halcrow, 2009)[1]: ISIS 1D models of the Tame catchment including major tributaries which have been included in the model as short reaches (such as the River Rea) and point inflow such as the River Ford, and Hockley. This model was built upon previous studies with the most recent being 2005. This modelling was divided into two separate models, these being the upper Tame and the lower Tame with the upper most extent being Ashes Road, Oldbury and the downstream extent County Bridge, Willenhall. The Proposed Scheme will fall within the upper Tame model's extent.

2.3.3 Environment Agency Midlands Region Central Visualisation modelling, (Halcrow, 2011)[2]: This model was based on the 2009 SFRM model[1], and has been converted to an ISIS-TUFLOW (1D/2D) model with the TUFLOW component representing the floodplain using a 10m regular grid. This modelling was divided into three separate models, these being the upper Tame, middle Tame and lower Tame, the Proposed Scheme will fall within reach of the Lower Tame model.

[1] River Tame Flood Risk Mapping Study, 2009, Environment Agency Midlands Region, Halcrow Group Ltd, Final Report, April

[2] SFRM2 Flood Visualisation Study –Central Area, 2011, Environment Agency, Draft Project Report, Halcrow, September

2.3.4 LiDAR (GeoStore, 2012): LiDAR ground elevation data with 1m resolution (typically ±0.15 tolerance on elevations). The LiDAR data for the entire area of interest was surveyed in 2008. The data was provided in two formats: 1) digital terrain model (DTM) data representing a bare earth scenario and 2) digital surface model (DSM), which includes features such as buildings and tree canopies.

2.3.5 Ordnance Survey MasterMap (Ordnance Survey, 2012) - MasterMap (vector) data features in the MasterMap data for the entire area of interest were last verified between 2001 and 2012.

2.3.6 Ordnance Survey raster mapping provided at 10k, 25k, 250k scales.

2.3.7 Aerial imagery (Geostore, 2012) provided at 25cm resolution and of varying collection dates prior to 2012.

2.3.8 Topographic survey data - no topographical survey data was available.

2.3.9 Environment Agency flood map data -Flood Zone 2, Flood Zone 3, defences, storage areas.

2.3.10 Historic flood extents - historic flood extents were provided for the 1992 and 2007.

2.3.11 Hydrometric data: Water Orton gauged water levels and flows for January 1999 and November 2000 from the Halcrow 2009 study.

3 Hydrology

3.1.1 The derivation of the inflow hydrology undertaken as part of the Environment Agency's 1D ISIS SFRM model (2009)[1] has been based on the following process:

- construction of a rainfall runoff model to generate hydrographs that reflect the performance of the catchment (at a particular location) to rainfall events across different storm duration (7.75, 10.75 hours and 15.75 hours);
- calibration of the modelled hydrographs using observed rainfall and flow data (using multiple gauging stations and rainfall gauges across the catchment);
- statistical analysis of observed data to generate peak flows for the full range of return period flood events (using multiple gauging stations) ; and
- incorporation of peak flows to modelled hydrographs to generate peak flow hydrographs for return period flood events.

3.1.2 This process of flow derivation matches current best practice to generate robust flow estimates.

3.1.3 This inflow hydrology has then been simulated within the 1D-2D ISIS TUFLOW visualisation model (2011)[2]. The results of this model have then been compared to the performance of the 1D ISIS SFRM model in terms of the peak flow at Water Orton gauging station during the 5% AEP and 0.1% AEP. This comparison indicates a consistent level of performance between the two models at Water Orton. Full details are given in the hydrology technical note appended to this report. The full range of return period flood hydrographs have been utilised within the model without amendment, including the median flow (Qmed) and the 1% AEP plus an appropriate allowance for climate change (CC).

3.1.4 The magnitude of the inflows introduced into the model for the three storm durations are indicated in Table 1, Table 2 and Table 3.

Table 1: Inflows to the model (7.75 hour duration)

	AEP (%)								
Location	**Qmed**	**20**	**10**	**5**	**2**	**1.33**	**1**	**1 +CC**	**0.1**
Tame	56	63.2	69	102.9	123.3	127.8	133.9	145.2	193.8
Rea	18.2	23.8	28.5	34.7	45.2	50.7	85.0	102.0	165.3
Walmley	9.8	12.9	15.5	19.2	25.7	29.1	49.0	58.8	100.6

Table 2: Inflows to the model (10.75 hour duration)

	AEP (%)								
Location	**Qmed**	**20**	**10**	**5**	**2**	**1.33**	**1**	**1 +CC**	**0.1**
Tame	50.5	60.3	66.6	103.5	124.7	128.5	133	142.7	191.2
Rea	16.7	21.6	25.9	31.5	40.5	45.2	75.3	90.4	142.2
Walmley	9.9	12.9	15.7	19.6	25.8	29.1	48.6	58.3	97.0

Table 3: Inflows to the model (15.75 hour duration)

	AEP (%)								
Location	**Qmed**	**20**	**10**	**5**	**2**	**1.33**	**1**	**1 +CC**	**0.1**
Tame	50.8	56.3	64.2	102.4	122	124.8	130.2	139.3	188.4
Rea	14.7	18.7	22.6	27.1	34.4	38.2	63.9	76.6	117.3
Walmley	9.7	12.5	15.5	19.0	24.7	27.7	46.7	56.0	90.7

4 Modelling approach and implementation – baseline model

4.1.1 Two hydraulic models were provided by the Environment Agency that included the reach of the River Tame adjacent to the Proposed Scheme. The first model is an ISIS 1D, model which was developed in 2009 for the purposes of Strategic Flood Risk Mapping[1]. The second model is an ISIS-TUFLOW 1D-2D model which was developed in 2011[2] and based on the ISIS SFRM model and was updated for the purposes of flood visualisation. A review of both models was undertaken by Arup (Arup, 2012)[3] as part of this study.

4.1.2 Given the urban location of the River Tame in the vicinity of HS2 and the potential complex overland floodplain flow routes, it was agreed that an ISIS-TUFLOW 1D-2D model would be more appropriate than an ISIS 1D model for assessing flood risk. The existing ISIS-TUFLOW visualisation model of the River Tame was therefore selected and amended based on the findings of the review.

4.1.3 However, there are limitations to this, including uncertainty associated with the input hydrology data as this analysis was not undertaken herein. No recent channel survey has been undertaken and therefore any changes in geometry of the channel or structures in the channel are not represented. Various survey data sets have been used in the development of the model dating from 1987 through to 2006, with a crest level survey in Bromford undertaken in 2007. A comprehensive and detailed review of model parameters such as roughness and weir coefficients on structures has not been reviewed. However, sensitivity analysis on model parameters has been undertaken.

4.1.4 The model starts approximately 1.1km upstream of the confluence of the River Rea, and inflow data has been extracted from the 2009 ISIS model as an inflow boundary. The downstream boundary is approximately 2.7km downstream of the start/end of the Castle Bromwich and Bromford area (CFA25).

4.2 Model review

4.2.1 The 2009 ISIS model[1] and the 2011 ISIS-TUFLOW model[2] were reviewed to determine their suitability for use in this study and to identify potential modifications that could be made to improve their representation of flood risk. The reviews covered model data, schematisation and runtime performance and are detailed elsewhere (Arup, 2012)[3]. The 2011 ISIS-TUFLOW model[2] of the River Tame was considered to represent flood risk more accurately, particularly in urban areas of the floodplain where it better represents complex flow paths. This model also showed better runtime performance than the 2009 SFRM model[1], which provides greater confidence in model results. For these reasons, the 2011 ISIS-TUFLOW model[2] was taken forward for further development in this study.

[3] Model review - River Tame SFRM upper model and Model review - River Tame visualisation lower model, Arup 2012

4.2.2 It was noted that there were large differences in water level between the 2009 and 2011 study. The maximum water level results from the visualisation model were compared against those from the SFRM model for the 0.1% AEP event (7.5hr storm duration) which shows the visualisation model generally gives slightly higher water levels than the SFRM model upstream of the Bromford area. The SFRM model consistently gives higher water levels downstream from this point with significant differences of up to 1.0m near the Parkhill Wood area. These were investigated but could not be resolved within this study.

4.2.3 Following the review, amendments were made to the 2011 ISIS-TUFLOW model to incorporate more recent ground level data and to improve the representation of flows, as described below.

4.3 TUFLOW model component

Cell size, topography and extent

4.3.2 The originally supplied ISIS-TUFLOW model had a grid size of 10m, with was sufficient for the purposes of visualisation of the flood response. However, for this study the grid size was reduced to 6m to improve the accuracy of floodplain flow routes. The ground elevations in the model were updated using the most recent 1m LiDAR DTM data, which was flown in 2008. The model extent was retained from the original supplied data.

4.3.3 The bank levels in the existing model were retained for most of the reach as filtering issues identified in the LiDAR limited its use to improve the representation of these. However, onsite observations around the Park Hall nature reserve suggested that there had been a failure of the embankment which was therefore included within the model. Furthermore at the upstream extent of Park Hall nature reserve there is a deliberate lowering of banks which was also included within the baseline model.

4.3.4 Some stability issues were identified in parts of the TUFLOW model due to irregular topography and also in some areas along the interface between the ISIS and TUFLOW models due to unrealistic recirculation patterns. These were resolved by either increasing the roughness in the TUFLOW model or smoothing out the topography at the areas of instability.

Land use and roughness coefficients

4.3.5 Roughness values are specified in the TUFLOW model spatially and assigned a Manning's n roughness coefficient. The 2011 model used a constant roughness value of 0.05, which is considered an over-simplification and does not represent the increased resistance to flow through and around buildings. The Ordnance Survey MasterMap data was therefore used to spatially define land use in the model and roughness values were specified for each (see Table 4).

Table 4: Roughness values used in TUFLOW domain

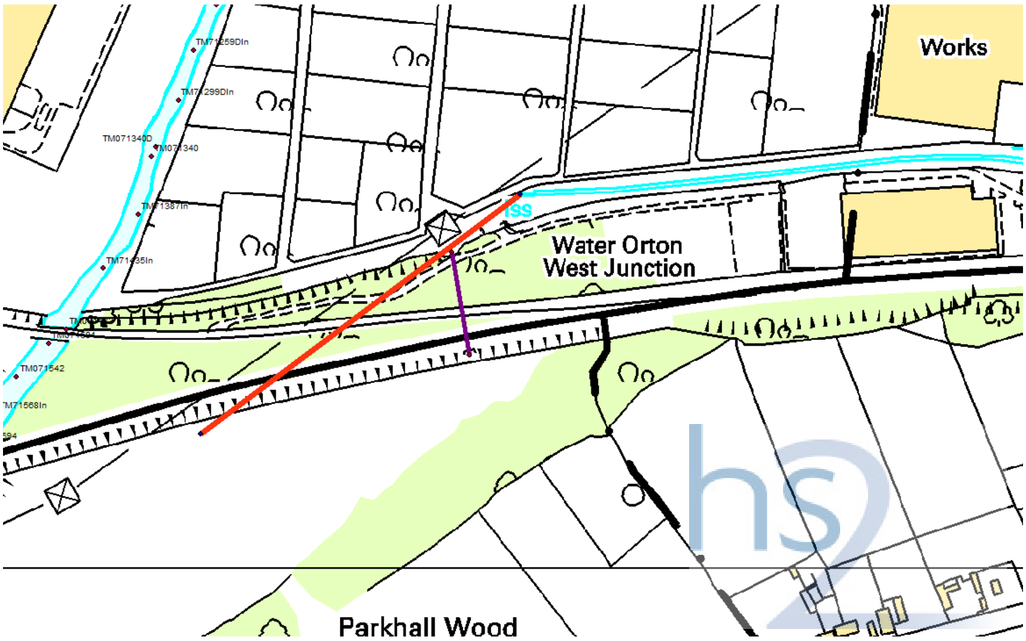
Material	Manning's n	Material	Manning's n
Manmade surface	0.05	Land (General Surface - Step)	0.05
Garden	0.2	Structures	0.04
Path or track	0.035	Land	0.05
Railway	0.05	Structures	0.04
Road	0.025	Land	0.05
Buildings	0.3	Land	0.05
Glasshouses	0.3	Road Track and Paths	0.035
Trees	0.07	stability	0.07 to 0.25
Water	0.03	Fields and natural land	0.05

1D-2D connection lines

4.3.6 ISIS and TUFLOW are dynamically linked using water level and flow boundaries that connect the ISIS 1D model nodes to the TUFLOW 2D model representing the floodplain. Only minor updates to slightly improve the alignment of these boundaries were carried out for this study.

4.3.7 The existing TUFLOW model includes a culvert under Network Rail into the Water Orton flood relief channel. The culvert is modelled in the ESTRY 1D component of the TUFLOW software. The location of this culvert in the model was revised following a review of LiDAR and aerial photography. The culvert length in the model was also reduced from 300m to 78m. Figure 3 shows the original culvert location (red) and the revised culvert location (purple) in the model.

Figure 3: Location of Water Orton flood relief culvert in central visualisation model (2011) and updated (2013) modelling



ISIS model component

Model geometry

4.3.8 Modifications to the ISIS model were limited to the following updates.

4.3.9 Three spill units representing flow bypassing over bridge parapets and other structures were identified as being too long and representing bypass flows already represented in the TUFLOW component of the model. This results in a 'double counting' of the bypass flow, which can cause upstream water levels to be under-estimated. These spill units were therefore shortened to only include the bypass flows within the ISIS component of the model.

4.3.10 Three weir coefficients were increased from 1.0 to 1.2 to allow for improved representation of losses around the motorway column piers. This modification is likely to have a relatively small impact on water levels during flood conditions.

4.3.11 The initial conditions supplied with the 2011 model were used initially within this study but were found to be inconsistent with the initial flows. A test was carried out for the 50% AEP and 1% AEP plus climate change events to determine if improved initial conditions would have any effect on peak water levels. It was found that the initial conditions supplied had no effect on peak levels and therefore the supplied data were used. However, for completeness, the initial conditions were improved by making them consistent with the initial flows in the model.

4.3.12 A pipe connection between the River Tame and the Park Hall nature reserve was identified during a site inspection (Figure 4). The diameter was measured as 600mm a with an invert level of approximately 77.6m AOD. This pipe connection was added to the ISIS model using an outfall unit that is dynamically linked to the TUFLOW model at the pond.

Figure 4: Pipe connection at Park Hall nature reserve

Boundary conditions

4.3.13 There are a number of tributaries to the River Tame in the area of interest, with the main one being the River Rea. In addition to this there is also the Plants Brook and Dunlop Channel. The main upstream boundary in this model is a flow time boundary which has been extracted from the full SFRM 2009 ISIS model. Inflow boundaries for the tributaries have been included as flood estimation handbook, FEH, boundaries.

4.3.14 The downstream boundary is a normal depth boundary with a bed slope of 0.002m/m as per the supplied model data.

5 Baseline modelling results and model proving

5.1 Simulations carried out

5.1.1 Flood events for a range of return periods have been simulated as part of this modelling exercise to determine the baseline flood risk for the 50%, 10%, 5%, 2%, 1.33%, 1% and 0.1% AEP events. A 1% AEP event with 20% increase in flows to account for climate change has also been simulated. Each return period has been simulated for 3 storm durations, these being 7.75 hours, 10.75 hours and 15.75 hour. The maximum water levels, depths and flood extents for all three storm durations have been taken for the purposes of assessing flood risk.

5.2 Summary of baseline results

5.2.1 Typically, the 7.75 hour storm duration gave the greatest flood extent, and peak flood level. However downstream of Park Hall nature reserve, the 15.75 hour storm duration resulted in, on occasions and in discrete areas, slightly greater flood extents and level. To ensure that at all times the highest level and greatest extent was used, the maximum of all storm durations were used for the purposes of flood mapping.

5.2.2 The results show that for the 50% AEP event typically in-bank flow is observed, with the exception of Park Hall nature reserve where the scour hole in the embankment causes flooding in this area but it is confined to the immediate vicinity around the pond. At the 10% AEP event and above, the flooding (via the scour hole and culvert connection) into Park Hall nature reserve spreads downstream towards the flood relief culvert under Network Rail and into the Water Orton flood relief channel. There is additional flow into this channel from the main upstream inlet adjacent to the River Tame immediately downstream of Network Rail. By the 5% AEP event, there is flooding within the Bromford area around the Tame Valley Primary School and along Chillinghome Road and Bromford Drive. There is also overtopping of the River Tame immediately downstream of Network Rail at Water Orton. At the 2% AEP and 1.33% AEP events, there is widespread inundation of Bromford, Park Hall nature reserve and the Water Orton district. Upstream of Bromford, flows are still confined to the channel. By the 1% AEP event, the areas upstream of Bromford start to flood. During the 1% AEP plus climate change scenario, the flooding on the northern side of railway at Bromford is more significant, with flow in the Dunlop Channel which connects back into the River Tame at the western extent of Park Hall nature reserve. There is also flooding in to the Castle Vale district of Birmingham. At 0.1% AEP, much of the extent model is flooded.

5.3 Implications for the Proposed Scheme

5.3.1 Proposals include the Washwood Heath depot, a major depot within the Washwood Heath area, the modelling indicates that for events of 1% AEP or more frequent there is no inundation of this area. At the 1% AEP plus climate change event there is backflow into the Washwood Heath drainage channel which drains to the Tame in the upstream Bromford area. Please refer to Volume 5: Map book WR-05 and WR-06 for extent of flooding within this channel for the 5% AEP and 1% AEP plus CC events. It should be noted that the existing Network Rail infrastructure within this area is overtopped at the 1% AEP plus climate change event.

5.3.2 The proposed Bromford tunnel east portal will be in the vicinity of Orton Way. The baseline model indicates no flooding within this area at the 1% AEP plus climate change event, however, there will be flooding around the confluence of Dunlop Channel and the River Tame, close to the Bromford tunnel west portal and to the Proposed Scheme in this vicinity.

5.3.3 The Proposed Scheme will continue through Park Hall nature reserve on a variety of embankment and viaduct. Currently this area experiences frequent flooding. It is proposed that the River Tame will be diverted in this area from its engineered straight channel to an alternative channel adjacent to its original alignment.

5.4 Validation

5.4.1 Given the scope and constraints of this stage of the study and given the lack of available observed flood event data, a full calibration exercise has not been undertaken. However to provide confidence in the updated hydraulic model, a validation exercise has been undertaken. Model flow boundaries for the 2009 ISIS model representing the following two historic flood events were provided by the Environment Agency (generated by Halcrow[1]):

- November 2000
- January 1999

5.4.2 These were derived by Halcrow using observed available rainfall and river gauge data as part of the SFRM study (Halcrow, 2009). These mode boundaries have been simulated using the original 2009 ISIS model and the flow results have been extracted and used to define the upstream inflow boundaries to the updated ISIS-TUFLOW model. These flows have been routed through the updated ISIS-TUFLOW model and the model results have been compared against the available gauge data provided in the SFRM study (Halcrow, 2009).

5.4.3 The available gauge data was limited to the Water Orton gauge located at grid reference 416939, 271429). Table 5 compares the observed gauge data with the original 2009 1SIS model as well as the ISIS-TUFLOW model updated as part of this study. The results show that the model over predicts flow and water level for both flood events. However, it is noted that the results from the updated baseline model correlate to the gauged data better than to the Halcrow 2009 SFRM model.

Table 5: Comparison of observed and model data

		Observed	Arup, 2013	Absolute difference between 2013 model and observed
January 1999	Flow (m^3/s)	52.756	71.542	18.79
	Stage (mAOD)	76.418	76.776	0.36
November 2000	Flow (m^3/s)	85.01	92.957	7.95
	Stage (mAOD)	76.786	77.17	0.38

5.4.4 The information on the Environment Agency's Hi Flow website indicates that Water Orton gauge is a relatively reliable crump weir structure that can be used to derive flows up to the median flood (Qmed). In excess of Qmed, there is no instantaneous flow gauging data to validate the station's performance and there is a possibility that water can overtop locally, which reduces the confidence in the flow data from this gauge.

5.4.5 The January 1999 event can be used to validate the model with a high level of confidence as the recorded and modelled flows are both less than the calculated Qmed. However, the level of confidence attributed to validation of the November 2000 event is lower due to the fact that the recorded and modelled flows are in excess of the calculated Qmed.

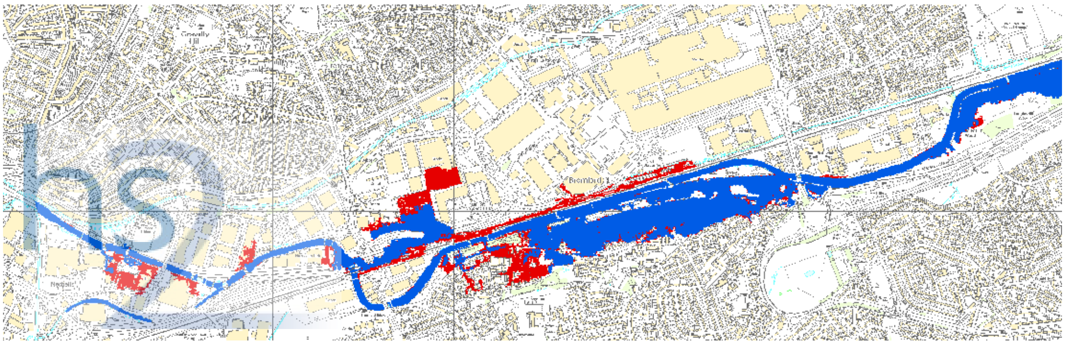
5.5 Sensitivity analysis

5.5.1 In addition, a range of sensitivity tests have been undertaken to understand the impact of uncertainty in model parameters on flood risk. The sensitivity analyses include:

- Increase/decrease Manning's roughness by 10% in ISIS domain;
- Increase/decrease Manning's roughness by 10% in TUFLOW domain; and
- Increase/decrease downstream water levels by 0.2m.

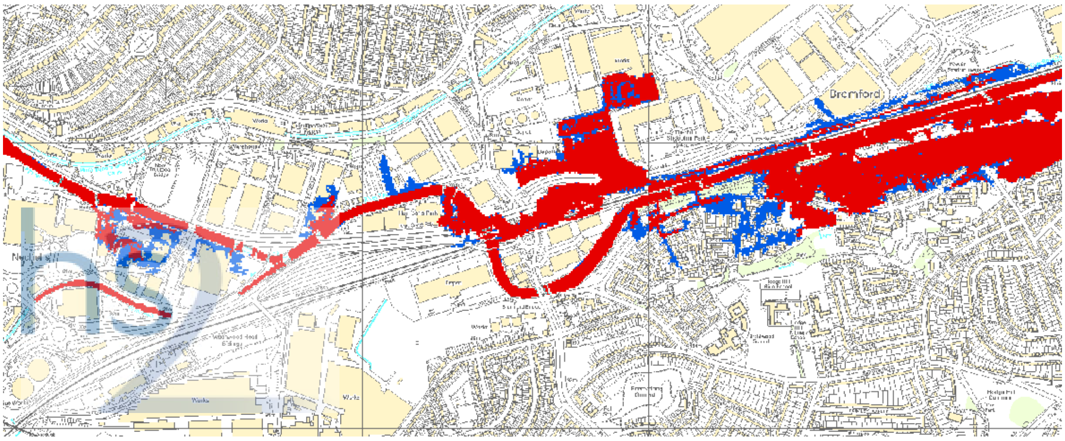
5.5.2 Increasing roughness within the ISIS domain by 10% resulted in average change in water level between the three storm durations of <0.1m, with a maximum change of 0.197m which was observed on the 15.75 hours storm duration at the upstream extent of the model by Walker Drive. The flood extents are larger in extent, particularly in the upstream around Bromford, the Fort Shopping Park and Gravelly Industrial Park. This can be seen in the figure below which shows the 1% AEP results (storm duration 15.75 hours) baseline in blue and sensitivity in red.

Figure 5: Comparison of increasing Manning's roughness by 10% in ISIS for 1% AEP, 15.75 hours storm duration



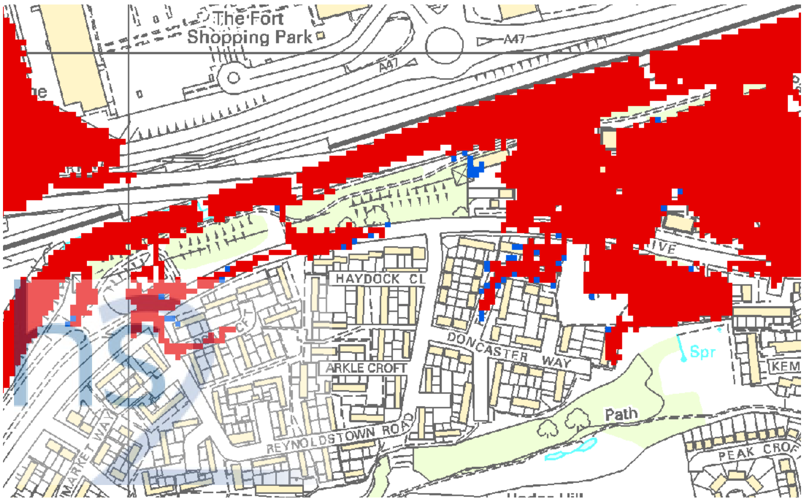
5.5.3 Decreasing roughness within the ISIS domain exhibited similar trends with the average change being <0.1m, with a maximum change of -0.202m observed at TM070097 at the downstream extent of the Water Orton flood relief channel. The flood extents are smaller than the baseline results around the Bromford, Gravelly Industrial Estate and Fort Shopping Park areas as shown below for the 1% AEP event (storm duration 7.75 hours).

Figure 6: Comparison of decreasing Manning's roughness by 10% in ISIS for 1% AEP, 7.75 hours storm duration



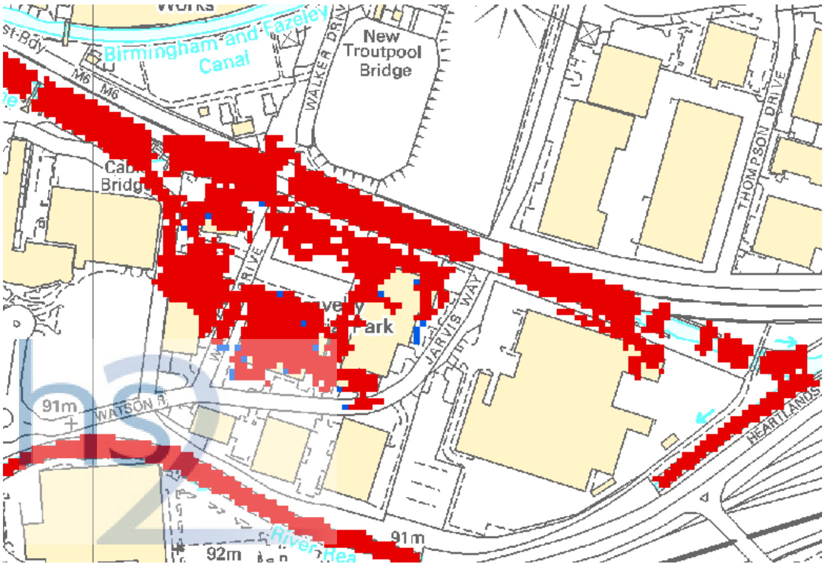
5.5.4 Decreasing roughness within the TUFLOW domain resulted in minimal change in peak water levels with a maximum difference of -0.027m for the 15.75 hours storm duration and an average change of 5mm across all durations. There is marginal difference between the baseline and sensitivity flood extents. The most noticeable change is in the Bromford area, and is shown below for the 15.75 hours duration below:

Figure 7: Comparison of decreasing Manning's roughness by 10% in TUFLOW for 1% AEP, 15.75 hours storm duration



5.5.5 Increasing roughness within the TUFLOW domain resulted in minimal change in peak water levels with a maximum difference of 0.027m for the 10.75 hours storm duration and an average change of 6mm across all durations. There is marginal difference between the baseline and sensitivity flood extents. The most noticeable change is in the Gravely Industrial Park, and is shown below for the 10.75 hours duration below:

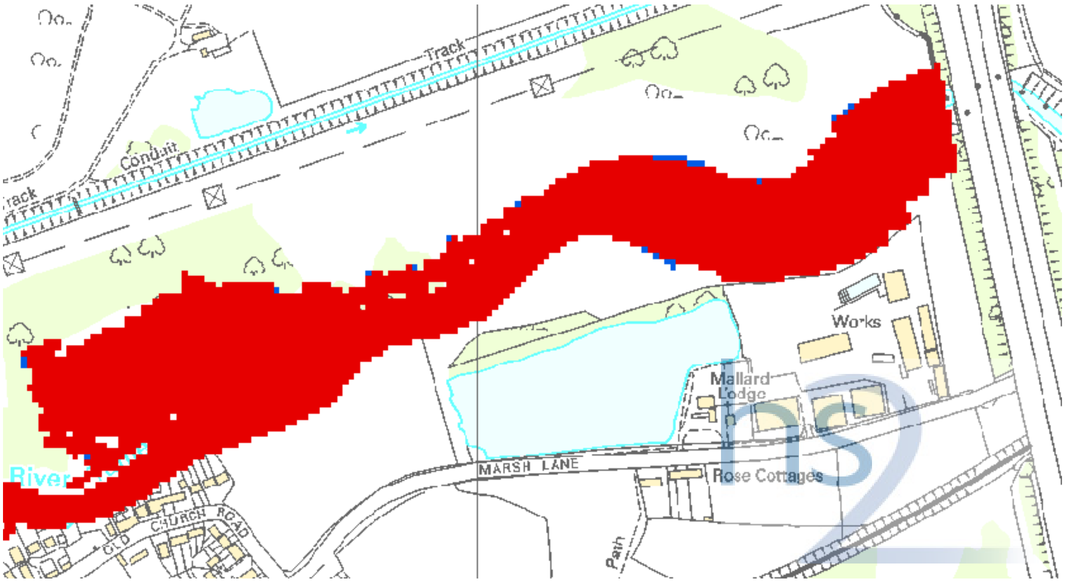
Figure 8: Comparison of increasing Manning's roughness by 10% in TUFLOW for 1% AEP, 10.75 hours storm duration



5.5.6 To assess the sensitivity of the model to changing downstream water levels, the stage-time results were extracted for the most downstream node for each storm duration, and 0.2m was added/subtracted from this to define the downstream water levels used in these sensitivity tests.

5.5.7 Decreasing downstream water levels by 0.2mm resulted in an average difference of 4mm, and likewise increasing the downstream water level the average increase was 5mm. In both cases the effect on the river as a whole from changing boundary conditions diminished to less than 0.01mm by 1km upstream. There were only marginal differences in flood extent for decreasing the boundary condition by 0.2mm in the 15.75 hours storm duration event (Figure 9).

Figure 9: Comparison of decreasing the downstream boundary by 200mm for 1% AEP, 15.75 hours storm duration



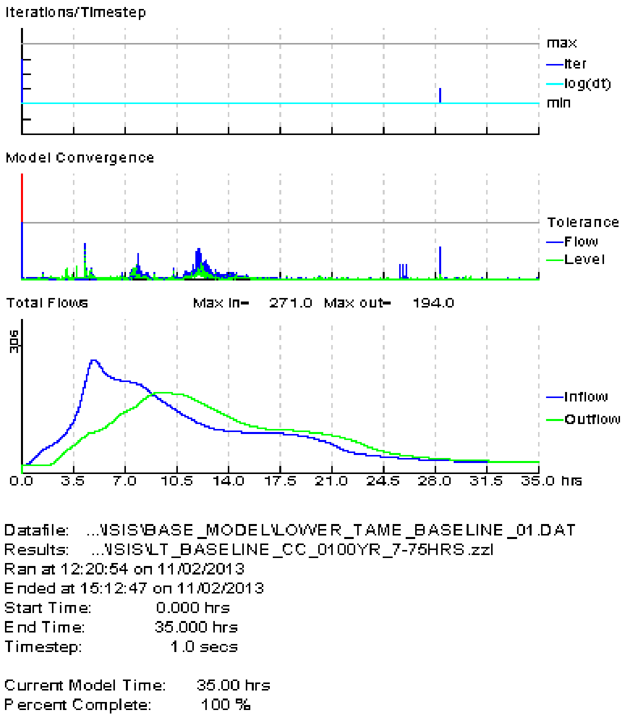
5.6 Run time performance

5.6.1 Run-time parameters were largely left at default values. The TUFLOW timestep was set to 2s and the ISIS timestep set to 1s, these values are appropriate for the 2D grid cell size of 6m. The 'dflood' parameter was set to 5m in the original model provided and this has been retained in the updated baseline model. The maximum number of iterations in ISIS was set to the default value of 6 and with no non-convergence problems, this indicates a well performing model.

5.6.2 Run time performance is an indication of the health of a model. For the baseline simulations, the typical error for mass balance within the TUFLOW domain is less than ±1%, with the exception of the 1% AEP event which is between -1 to -2%, with errors around the flood peak of approximately <±0.5%. Given the average mass error of all simulation is ±1%, the TUFLOW component of the model does not indicate mass balance problems.

5.6.3 The ISIS 1D run performance also indicates a stable, healthy model. With the exception of the first time step in the model simulation, non-convergence to the default flow and level tolerances does not occur throughout the simulation. A typical run performance log is shown below (1% AEP plus climate change, 7.75 hours storm duration).

Figure 10: Typical ISIS run log



5.6.4 The ISIS model time series results have been inspected and show no significant anomalies or oscillations in water levels or flows. This also indicates good model stability and provides confidence in the runtime performance of the model and the results.

6 Diversion channel within Park Hall nature reserve

6.1 Diversion options considered

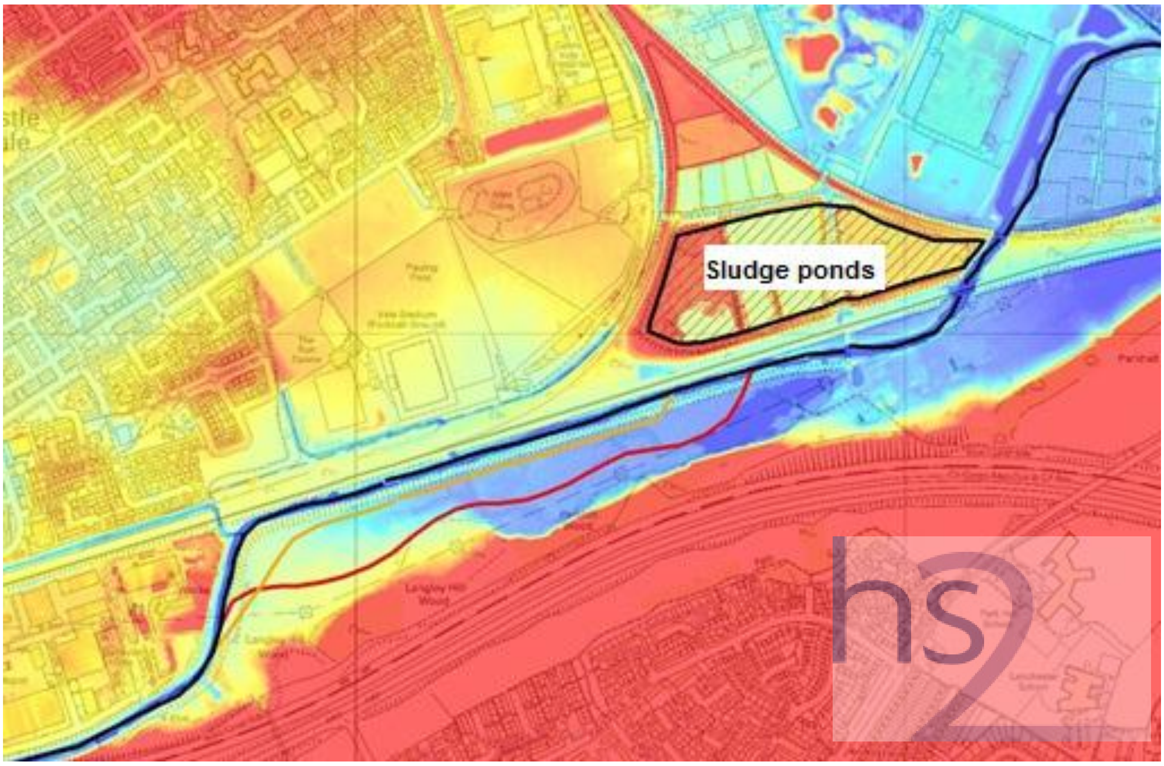
6.1.1 The Proposed Scheme will be designed to head towards Birmingham crossing the Park Hall nature reserve near the M6 motorway at Water Orton before running along the existing River Tame alignment into a tunnel. The Proposed Scheme is illustrated in Volume 2: Map book CT-06. In order to achieve the horizontal alignment of the Proposed Scheme, the River Tame at Park Hall nature reserve must be diverted. Several diversion options were considered as described below, with options 1 and 2 being development work which led to option 1 described within Section 2.6 of the Environmental Statement Volume 2 report. These are shown superimposed on the LiDAR ground level data in Figure 11.

- Option 1: meandering alignment through floodplain - bank levels will be set to existing ground levels (significantly lower than existing channel bank levels);
- Option 2: meandering alignment through floodplain - bank levels will be set to existing channel bank levels; and
- Option 3 (Option 4 within Volume 2 of the Environmental Statement Section 2.6): straight channel alignment adjacent to existing channel - bank levels will be set to existing channel bank level. For this option, the channel is split into three smaller channels towards the downstream end of the diversion and these are threaded through the River Tame viaduct piers. This will be the preferred alignment for the diversion channel.

6.1.2 Each of the above diversion options reduces overall floodplain volume, which may result in increased flood risk if mitigation measures are not employed. The Proposed Scheme will be designed not increase flood risk provided the correct implementation of all mitigation measures. This will be achieved by maintaining the overall flood storage capacity. Replacement floodplain flood storage has therefore been considered where necessary.

6.1.3 The above options were initially modelled (without any replacement floodplain storage) to assess the impact on flood risk. Where required, options to mitigate increase in flood risk through replacement floodplain storage have been developed and modelled.

Figure 11: Diversion options 1 and 2 (red) and option 3 (orange). Ground levels are blue (lowest) to red (highest)



6.2 Diversion model development

6.2.1 An ISIS-TUFLOW model was developed from the baseline model for each diversion option.

6.2.2 The ISIS model cross-sections representing the River Tame parallel to the diversion channel were modified to represent the proposed diversion channel geometries, which has been defined as:

- constant channel bed slope along diversion channel;
- channel bed width = 14m;
- channel side slope = 1 in 2.5;
- embankment side slope (landward side) = 1 in 3 (option 2) and 1 in 2.5 (option 3); and
- embankment crest width = 2m (option 2) and 2m on the right bank and 12m on the left bank to provide vehicular access (option 3).

6.2.3 The distance between cross-sections along the diversion channel reach was slightly reduced to accurately represent the length of the diversion channel. For option 1, the bank levels were set to the adjacent floodplain level as defined by the LiDAR DTM data. For options 2 and 3, the bank levels were set to the same as those within the existing baseline model.

6.2.4 For option 3, the geometry of the threaded channel through the viaduct piers towards the downstream end of the diversion channel as well as the adjacent viaduct piers were added to a bridge unit in the ISIS model. The high degree of skew of the piers relative to the direction of flow was represented by manually scaling the threaded channel geometry by a factor of 0.3. However, it should be noted that 1D modelling cannot accurately represent the complex hydraulics at such a structure and 3D or physical modelling would be required to accurately assess the impact of the piers and the tight bend in the channel.

6.2.5 The TUFLOW components of the models were amended to incorporate the new alignments of the diversion channels. This included representing the footprint of the channel and channel embankments (options 2 and 3) and realigning the 1D-2D model connections and bank elevation lines.

6.2.6 For option 1, initial model testing showed instability along part of the diversion channel caused by artificial recirculation. This instability was resolved by using a material layer with a slightly higher roughness covering the 2D grid cells adjacent to the 1D-2D model connections along the diversion channel. The roughness was increased from 0.05 to 0.07, though a value of 0.1 was later used for the 2% AEP and 1.33% AEP simulations to resolve instability.

6.2.7 The small volume of floodplain lost due to the proposed viaduct piers was represented in the TUFLOW component of the model by effectively deactivating the grid cells associated with these. It is assumed the River Tame viaduct piers will be 2m x 12m with spacing of 25m. Given the grid size of the TUFLOW model is 6m, only every third viaduct pier was represented to represent the correct volume of floodplain that would be lost.

6.3 Impact on flood risk

6.3.1 The diversion channel option models were initially used to simulate various flood events without any replacement floodplain storage to understand the impacts on downstream water levels without any mitigation in place.

Option 1:

6.3.2 Model results show an increase in downstream flood risk. For the 1% AEP + CC event, a maximum increase of 28mm was observed in the channel downstream of the diversion and flood extents were found to be very slightly larger downstream. Floodplain depth was found to be 90mm deeper at a 2 Hectare area near the Sewage Works adjacent to Water Orton Lane. The results show that due to the lower bank levels, natural floodplain storage in the Park Hall nature reserve is used less efficiently than in the baseline model because more volume is used up earlier in the flood event leaving less storage capacity during the peak of the flood event.

6.3.3 It is possible to attenuate this increase in flood levels by providing offline storage at the sewage works, an overview of the potential for this site to provide storage is provided in Annex B of this report.

Option 2:

6.3.4 Model results show a maximum increase in water level of 11mm in the channel downstream of the diversion for the 1% AEP +CC event, with peak flow slightly increased downstream of Park Hall nature reserve. Flood extents are very slightly larger downstream but this is limited to a small number of individual grid cells. Floodplain depth was found to be 30mm deeper at a 2 hectare area near the sewage works adjacent to Water Orton Lane. Given the increase in downstream depths and extent is very small, it is likely that replacement floodplain storage could be used to mitigate the small increase in downstream flood risk. It was decided to park this option and focus on options 1 and 3.

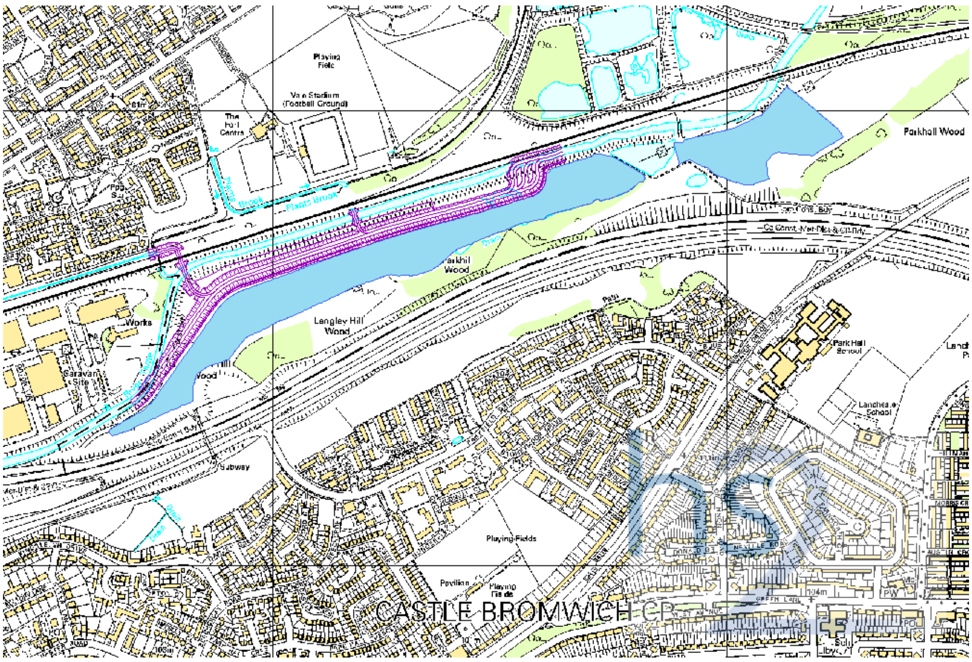
Option 3:

6.3.5 The model results show an increase in downstream water levels and flood extents for the 1% AEP + CC event locally within the Park Hall nature reserve. The maximum increase in water level, for the 1% AEP +CC event (7.75 hours storm duration) is approximately 20mm within the channel downstream of the diversion, but floodplain depth increases of approximately 100mm were noted in a few isolated locations within the industrial area south of Water Orton Lane. The downstream water levels have been increased compared to the baseline model as the earthworks required to facilitate the diversion reduce floodplain storage. This option will be further developed to include replacement floodplain storage within the Park Hall nature reserve.

6.4 Replacement floodplain storage

6.4.1 To mitigate the increase in flood levels and peak flow downstream of Park Hall nature reserve, two areas have been identified within Park Hall nature reserve to provide replacement floodplain storage. These are shown in Volume 2: Map book CT-06. This has been demonstrated using the hydraulic model as discussed in the following chapter.

Figure 12: Proposed area of replacement floodplain storage within Park Hall nature reserve



6.4.2 LiDAR was used to ascertain the water level within the existing pond within Park Hall nature reserve, and it was assumed that this is approximately groundwater level. The base level of the proposed replacement floodplain storage areas has been set to just above this level, but further data on groundwater is required, and once received, the sizing should be reviewed.

7 Post-development model

7.1.1 The post development model was developed from the baseline model and incorporates the Proposed Scheme (Volume 2: Map book CT-06) as follows:

- earthworks at Park Hall nature reserve: this was represented in the TUFLOW model using proposed earthworks ground levels provided in the Proposed Scheme drawings;
- earthworks at Washwood Heath depot: this was represented in the TUFLOW model using proposed earthworks ground levels provided in Proposed Scheme drawings;
- wall at Washwood Heath depot: the wall at the Washwood Heath depot will provide flood protection for the 0.1% AEP. The crest level of the wall was therefore set very high to prevent any overtopping in the 0.1% AEP. The alignment of the wall was provided in the Proposed Scheme drawings;
- new drainage channel at Washwood Heath depot: this was represented in the TUFLOW model using proposed earthworks ground levels for the channel provided in the Proposed Scheme drawings;
- River Tame viaduct piers at Park Hall nature reserve: the small volume of floodplain lost due to the proposed viaduct piers was represented in the TUFLOW component of the model by effectively deactivating the grid cells associated with these. It is assumed the viaduct piers are 2m x 12m with spacing of 25m, the location of these is shown in the Proposed Scheme drawings. Given the grid size of the TUFLOW model is 6m, only every third viaduct pier was represented to represent the correct volume of floodplain that would be lost.
- channel diversion at Park Hall nature reserve: straight alignment with bank levels set to existing bank levels. This is the preferred alignment for the diversion channel and its representation within the model is described Section 6.3 above); and
- replacement floodplain storage within the nature reserve involved excavating ground levels throughout much of the reserve to the pond in the Park Hall nature reserve (discussed in Section 6.3 above).

7.2 Results and analysis

7.2.1 The post development model was used to simulate return periods between 50% AEP and 0.1% AEP, each for 7.75, 10.75 and 15.75hr storm durations. The results show the post development situation with the replacement floodplain storage option developed to mitigate flood risk from the Park Hall nature reserve channel diversion does not cause an increase in risk to third parties for events up to the 1% AEP plus climate change event. Flood levels within Parkhall nature reserve increase by up to 30mm for the 1% AEP plus CC event. One specific return period/storm duration combination resulted in a small increase (up to 19mm) in downstream water levels and a marginal increase in flood extent which is due to improved conveyance through the diversion. However, flood risk has been assessed as the worst flood levels and extents across all three storm durations and when these flood levels/extents are compared to baseline, there is no significant change to flood risk. The design can be developed such that this increase is mitigated or managed by locally raising and lowering bank levels within the Park Hall nature reserve reach of the Tame. Across all return periods, there is a small (up to 30mm) increase in water levels within Park Hall nature reserve where the replacement floodplain storage area has been sought.

7.2.2 Flood extents for the maximum of all storm durations assessed for the baseline situation and the post development situation are provided in the FRA accompanying appendices for the 5% AEP and 1% AEP plus climate change events

7.3 Blockage analysis

7.3.1 Blockage analysis was undertaken on key structures to assess the impacts to the Proposed Scheme. The approach taken is summarised herein:

- bridges and culverts blocked by 10%; and
- viaducts blocked by 2%.

7.3.2 The structures determined as critical and the blockage scenarios are:

- blockage 1 - Network Rail crossing upstream of the A47 Heartlands Parkway;
- blockage 2 - A4040 Hagley Road;
- blockage 3 - CA452 hester Road Bridge; and
- blockage 4 – Network Rail downstream of Park Hall nature reserve, the Water Orton flood relief culvert and the River Tame viaduct.

7.3.3 These scenarios were ran for all three storm durations for the 0.1% AEP event, and the impact analysed by taking the highest of all blockage scenarios for all scenarios. Typically, the increase in water level immediately adjacent to blocked structures are in the range 110-250mm, however, blockage 2 resulted in increases of approximately 430mm at the structure. The blockage analysis flood levels were used to inform the level of protection for the Proposed Scheme. Therefore there are no residual risks to the Proposed Scheme provided all mitigation measures are implemented.

7.4 Recommendations

7.4.1 Based on the modelling undertaken, a solution can be found to divert the River Tame within the Park Hall nature reserve; however this involves excavation/ removal of land from the nature reserve.

7.4.2 This model has been developed on the interim preliminary design B4 condition and checked against CP2.

7.4.3 Additional surveys will be required to provide updated bank top and channel information within the vicinity of the scheme during later stages of design.

8 Conclusions and discussions

8.1.1 The River Tame has been subject to a number of modelling exercises over many years, the most recent prior to this study being in 2011. The 2009 and 2011 models have been reviewed for use within this study, and have been amended to include site observations on the behaviour of this river system and included within a linked ISIS-TUFLOW model.

8.1.2 Updates to the baseline model were undertaken based on site observations and more recent data such as land use and floodplain topography.

8.1.3 The baseline condition has been modelled for a range of return periods, and has been validated against recoded data. The results show that the channel overtops within the Park Hall nature reserve at the 50% AEP event due to the scour hole/embankment failure, and flooding commences in the Bromford area at the 5% AEP event. The Washwood Heath area is well protected, with flooding being observed to the north of Network Rail at the 1% AEP event.

8.1.4 The baseline model was further developed to include the Proposed Scheme as detailed in Sections 0 and 7. This also included a diversion of the River Tame within Park Hall nature reserve and replacement floodplain storage provided within the reserve. The results of the model indicate that no significant change in flood risk for the post-development situation are predicted subject to correct implementation of all mitigation measures.

9 References

Environment Agency, (2009), River Tame Flood Risk Mapping Study Environment Agency Midlands Region, Halcrow Group Ltd, Final Report, April

Environment Agency, (2011), SFRM2 Flood Visualisation Study –Central Area, Environment Agency, Draft Project Report, Halcrow, September

Ove Arup & Partners Ltd, (2012), Model review - River Tame SFRM upper model and Model review - River Tame visualisation lower model, Arup 2012

10 Annex A - hydrology review

10.1 Hydrology introduction

10.1.1 On the 13 June 2012, the Environment Agency's existing river hydraulic models of the River Tame were provided. These consisted of:

- a 1D ISIS Strategic Flood Risk Management (SFRM)[1] river hydraulic model developed to provide a catchment wide of perspective on flood risk. This model is referred to as the 'SFRM Model'. This is comprised of the upper and lower Tame components. The upper Tame includes the Wolverhampton and Olbury Arms of the River Tame which combine upstream of Bescot and extends to Water Orton. The lower Tame extends to the confluence with the River Trent at Alrewas; and
- an ISIS-Tuflow 1D-2D[2] river hydraulic model developed to aid visualisation of proposed flood risk management activities. This model is referred to as the 'Visualisation Model'. This model is comprised of three component parts (upper, middle and lower). The naming of the models are independent of the naming of the SFRM model. The upper model includes the upstream extent of the River Tame catchment as far Newton near Great Barr. The middle Tame covers Newton to Nechells and the Lower Model covers Nechells to Water Orton.

10.1.2 These models have been reviewed and updated for use in the design process in locations where the Proposed Scheme will interact with the River Tame.

10.1.3 An initial check has identified potential inconsistencies in the hydrological inputs into the models, and the flows observed in the models at the location of the proposals. Therefore, this note has been prepared which examines the hydrological inputs into the model and assesses whether additional hydrological calculations are required before the model can be used to set the baseline conditions and be used to assess River Tame diversion options.

10.2 Existing hydrology inputs

SFRM model[1]

10.2.2 The derivation of the input hydrology followed the following methodology:

- inflow catchments defined and checked using Flood Estimation Handbook (FEH) CD Rom and Ordnance Survey (OS) mapping sources;
- inflow catchment descriptors extracted from the FEH CD Rom; and
- inflows calculated using FEH rainfall runoff methodology for a range return period flows adopting default catchment descriptors .

10.2.3 The FEH rainfall runoff inflows were then calibrated and verified based on flow and level data collected during notable flood events which occurred in September 1994, January 1999, August 1999, June 2007 and November 2007. This data was collected at four gauging stations located along the main channel of the River Tame at (listed most upstream first):

- Bescot;
- Water Orton;
- Lea Marston; and
- Hopwas Bridge

10.2.4 The calibration process allowed the following catchment descriptors to be adjusted:

- standard average annual rainfall (SAAR) ;
- standard percentage runoff (SPR);
- baseFlow (BF);
- time to peak (Tp);
- following this a unit hydrograph for each event was developed which could then be used as a basis for the design flow hydrographs for each return period flow;
- FEH statistical method was then adopted to calculate peak flows for a range of return periods for the gauged locations named above;
- in the first instance the median flood (Qmed) was calculated for each gauging station location based on existing annual maximum series data;
- single site analysis was undertaken at each site and used to develop the growth curve required to derive flows for the 50%AEP, 20% AEP and 10% AEP flows;
- pooling group analysis was then used to derive the growth curves required to derive flows in excess of the 10% AEP flood;
- the rainfall runoff hydrological inflows method were then adjusted using scaling factors so that the flows calculated using the statistical method were achieved at the four gauging locations;
- design flows were then run through the model for a wide range of return periods including the 20%, 10%, 4%, 2%, 1.33%, 1%, 0.5% and 0.1% AEP; and
- analysis has undertaken to determine which duration generates maximum water levels within the SFRM model. This has involved running the model using different durations. Model simulations have been undertaken using 7.75 hours, 10.75 hours, 15.75 and 20.75 hours for the upper Tame and 15.75, 25.75 and 45.75 for the lower Tame. The 10.75 hours duration was selected as the critical duration in the upper Tame and 25.75 hours was selected in the lower Tame.

10.2.5 The process for determining the hydrological inputs for the River Tame appear to be consistent with current best practice as the method has developed an appropriate unit hydrograph based on observed events and statistical analysis has been used to determine peak flows. The unit hydrographs have then been scaled to match the calculated peaks. These scaling factors have then been applied to each inflow location.

10.2.6 However, there is some uncertainty regarding the River Rea inflow. The River Rea hydrology has been investigated as part of an independent study undertaken by Royal Haskoning. Investigating the River Rea model indicates that the inputs from the River Rea may be an underestimation.

10.2.7 Rainfall runoff and FEH statistical methods have been employed in a simple comparison study for the Rea catchment up to the Rea/Tame confluence. This 'quick' study indicates that the 0.1% AEP flow at the confluence for the River Rea is approximately $200m^3/s$. This is approximately $40m^3/s$ lower than the flow at calculated in the Tame model.

Visualisation model

10.2.8 The visualisation model of the Tame is constructed as three separate sections;

- upper Tame;
- middle Tame; and
- lower Tame.

10.2.9 The visualisation model does not represent the entire Tame catchment but instead is a representation of the metropolitan sections of the river. This means the downstream gauging stations at Lea Marston and Hopwas Bridge are not represented in the model.

10.2.10 The hydrological inputs have been based on the flows calculated as part of the SFRM model. However, only the 5% AEP and the 0.1% AEP flows have been introduced to the visualisation model.

10.2.11 The storm durations have been amended with a critical duration of 10.75 hours selected for the Upper Tame, 15.75 hours selected for the middle Tame and 7.75 hours selected for the Lower Tame.

10.2.12 The outflows from the upper Tame for each return period have been grouped together and then introduced at the upstream end of the middle Tame. The outflows for the middle Tame are then collected at the downstream end of the model and introduced at the upstream of the lower Tame.

10.2.13 In theory this approach should not cause any problems but the switch in hydraulic model from ISIS only to ISIS-Tuflow will have altered the routing properties of the floodplain. Since the approach adopted in the SFRM model is based on calibration at the river gauge locations, it is necessary to check that this calibration still holds true.

10.3 Result comparison

10.3.1 As the visualisation model does not contain the reach of the river downstream of Water Orton the calibration points at Lea Marston and Hopwas Bridge cannot be used in the hydrology check.

10.3.2 Both Bescot and Water Orton gauging stations are located within the confines of the upper Tame SFRM model.

10.3.3 Bescot is located within the upper Tame visualisation model and Water Orton is within the lower Tame visualisation model.

10.3.4 The scheme proposals are located within a reach of the river that corresponds to the location covered by the lower Tame visualisation model.

Peak flows

10.3.5 The peak flows generated for the 5% AEP and the 0.1% AEP at the two existing gauging locations represented in both models have been extracted from the SFRM and visualisation models. These are displayed in Table 6.

Table 6: Comparison of flows against Environment Agency models

AEP (%)	Gauging station	SFRM model (m^3/s)	Visualisation model (m^3/s)	Difference (m^3/s)	Difference (%)
5	Bescot	73.78	66.71	7.06	9.57
	Water Orton	119.05	126.56	7.51	5.93
0.1	Bescot	172.031	121.48	50.54	29.38
	Water Orton	256.75	268.25	11.50	2.8

10.3.6 This illustrates that the SFRM predicts higher flows at Bescot gauging station while the visualisation model predicts higher flows at Water Orton gauging station. This is not an unexpected discrepancy as the two gauging stations are located within different visualisation models.

10.3.7 The calculated differences between the two models indicate a moderate level of agreement at both gauging stations for the 5% AEP and indicate a good level of agreement for the 0.1% AEP at Water Orton.

10.3.8 However, there is a $50m^3/s$ variation between the flows predicted at Bescot for the 0.1% AEP between the two models. This represents a 29% difference and is significant.

River Levels

10.3.9 The water levels generated for the 5% and the 0.1% AEP at the two existing gauging locations represented in both models have been extracted from the SFRM and visualisation models. These are displayed in Table 7.

Table 7: Comparison of levels against Environment Agency models

AEP (%)	Gauging station	SFRM model (mAOD)	Visualisation model (mAOD)	Difference (m) SFRM - visualisation
5	Bescot	110.656	110.547	0.109
	Water Orton	77.496	77.473	0.023
0.1	Bescot	112.550	112.251	0.299
	Water Orton	79.456	79.023	0.433

10.3.10 This indicates that the water levels generated for the 5% AEP and 0.1% AEP simulations are within 0.5m of each other between the two models at the two gauging station locations. It also indicates that water levels are consistently lower within the visualisation model

10.3.11 The 5% AEP simulations have very good level of agreement at both Bescot and Water Orton gauging stations.

10.3.12 The difference (SFRM model level minus visualisation model) between the levels at Bescot for the 0.1% AEP flow is only 0.109m, which is well within normal modelling tolerances (normally 200 -300 mm). The level difference for the 0.1% AEP flow is 0.433m at Water Orton, this is in excess of normal tolerances. However, this difference is likely to be related to the routing of the water through the model and is not necessarily related to the hydrological inputs.

Peak flows and levels

10.3.13 When peak flow and levels are considered together there is consistency in the results at Bescot gauging station with the SFRM model generating higher flows and levels.

10.3.14 The flow derived by the SFRM model for the 5% AEP flood event is $7m^3/s$ greater and this equates to a 0.1m increase in water level.

10.3.15 The flow extracted from the SFRM model at Bescot gauging station is 50 m3/s greater than the visualisation model for the 0.1% AEP flood event. This results in a 0.299m increase in water level.

10.3.16 The consistency is not carried forward to Water Orton where the flows derived by the visualisation model are higher while the levels detected in the SFRM model are higher.

10.3.17 For the 5% AEP flood event the discrepancies are small with a 7 m^3/s difference in flow equating to a 0.02m difference in level. However, for the 0.1% AEP flood event the water level at Water Orton in the SFRM model is 0.433m higher than the visualisation model despite the flows being $11m^3/s$ lower.

Storm durations

10.3.18 The modelling reports that accompanied the SFRM model indicate that storm durations of 7.75hours, 10.75 hours, 15.75 and 20.75 hours were tested for the upper Tame. This resulted in duration of 10.75 hours being adopted as the critical event for the upper Tame SFRM model.

10.3.19 The storm durations reported to have been applied to visualisation models are based on the durations derived for the SFRM models with 10.75 hours applied to the upper Tame, 15.75 hours applied to the middle Tame and 7.75 hours lower Tame model.

10.3.20 This results in the same storm duration of 10.75 hours being applied at Bescot gauging station within both models.

10.3.21 At Water Orton, a duration of 10.75 hours is applied within the SFRM model, while a duration of 7.75 hours is applied within the visualisation model. However, within the visualisation model the flow has been routed through the middle Tame, with a duration of 15.75 hours being applied before the flow is routed through the lower Tame visualisation model. This complicates the relationship between the two models.

Other design flows

10.3.22 It has been decided that the visualisation model is the most appropriate to be used to establish the baseline river hydraulic conditions for the post development model which will include the Proposed Scheme. This model can then be used to assess the impact of the Proposed Scheme in terms of flood risk and ensure flood risk is not increased to adjacent development.

10.3.23 However, in its current form the model only contains the 5% AEP and 0.1% AEP flood events.

10.3.24 It will be necessary for all design flows generated as part of the SFRM model to be simulated.

10.3.25 This check confirms that the SFRM and Visualisation model have a reasonable level of agreement for the 5% AEP and 0.1% AEP events. As the flow hydrographs and peak flows for the 5% AEP and 0.1% AEP events have been generated in the same way as the other design flows (Qmed, 20%, 10%, 5%, 2%, 1% and 1% plus climate change) it is safe to adopt these other design flow hydrographs, scaling factors and peak flows without undertaking any additional analysis.

10.4 Conclusions

10.4.1 Although there are clear inconsistencies between the two models discussed, there is moderate consistency between the flow estimates generated at Water Orton for the 5% AEP event, and good consistency between flow estimates at the 0.1% AEP.

10.4.2 It is recognised that the water levels are lower for the visualisation model at Water Orton even though the flow is greater. However, a reduction in water level in a 2D model is not unusual and the water levels generated by the visualisation model are consistently lower than the SFRM model at these gauged locations. The difference in levels generated between the two models for the 0.1% AEP is just about acceptable.

10.4.3 As Water Orton is the gauging station closer to the study area it is far more important than any inaccuracies experienced at Bescot. Therefore, based on the consistency of the flow estimates at Water Orton it is considered appropriate to take this model and the existing hydrological inputs forward to create the baseline model of the River Tame for use in the post development model which incorporated the Proposed Scheme.

10.4.4 The flows for the full range of return period flows (Qmed, 20%, 10%, 5%, 2%, 1.33%, 1%, 1% plus 20% climate change allowance and 0.1% AEP) will all be applied to the visualisation model and introduced in the way described herein.

11 Annex B

11.1 Development of replacement floodplain storage at Severn Trent Water site

11.1.1 Modelling was undertaken to identify whether the increased flood risk resulting from channel diversion option 1 could be mitigated through replacement floodplain storage. Replacement floodplain storage areas have been considered at:

- small pockets of floodplain in the floodplain towards the upstream end of the diversion channel;
- all floodplain areas adjacent to the channel diversion and downstream to the Network Rail embankment; and
- wastewater treatment works sludge ponds (shown in Figure 11). This would be offline storage, which would drain back into the river after a flood event through an outfall pipe.

11.1.2 Replacement floodplain storage has been represented in the TUFLOW component of the model by defining the physical extent of the storage area and assigning associated invert levels (excavated levels) for these areas. The invert level of the modelled replacement floodplain storage has been set to approximately 0.5m above the normal water levels in the adjacent river channel to prevent ground water issues. For the floodplain adjacent to the diversion channel this varies between 77.8 and 78.2m AOD while for the sludge ponds this is taken to be 78m AOD.

11.1.3 For storage at the disuses water treatment works, which would require significant excavation (between approximately 2 and 6m depth) the storage area extent has been defined in the model as the invert of the storage area. The excavation for the storage area is represented in the model as having vertical sides, though in reality side slopes would be required. Consequently, the model slightly under-estimates storage at the disused ponds as the storage between the side slopes and the toe of the slopes is not represented. However, the purposes of the modelling are to inform outline design and the effect of incorporating side slopes will be relatively small.

11.1.4 The land required for the replacement floodplain storage at the sludge ponds would be greater than the extent used in the model as it would need to include the side slopes and space for access where necessary.

11.1.5 For modelling replacement floodplain storage at the sludge ponds, it has been assumed a spillway such as a reinforced grass bank, would be located immediately downstream of the National Rail line at the western end of the storage area. Initially this was represented in TUFLOW but this was subsequently changed to a spill unit in ISIS linked to the TUFLOW model. This arrangement allows more precise modification of the spillway crest profile, length and discharge coefficient.

11.1.6 The option 1 diversion channel model was initially used to simulate a range of flood events from the 50% AEP up to the 0.1% AEP for with a range of replacement floodplain storage options. The results showed that:

- Replacement floodplain storage within the floodplain adjacent to the channel diversion is

relatively ineffective. Even when using all available floodplain area, flood risk is still increased downstream. This is largely because the storage is used up earlier in the flood event leaving less storage capacity during the peak of the flood event. This option also causes higher flows through the culvert under the Network Rail embankment for lower return periods. These replacement floodplain storage options were discounted in favour of storage at the sludge ponds;

- Replacement floodplain storage at the sludge ponds is effective at mitigating the increase in downstream flood risk for larger floods, e.g. 1% AEP. Using a relatively high spillway crest level increases the efficiency of the storage as less storage is wasted before the peak. The spillway level could be optimised to minimise downstream impacts for a given flood event. Tests showed that when only considering larger floods, the extent of replacement floodplain storage could be reduced by using the storage efficiently through optimisation of spillway level;
- For smaller flood events, e.g. 1% AEP, where peak water levels are lower, the spillway crest level into the replacement floodplain storage area must be significantly lower than is optimal for larger events to enable the storage area to be used for smaller events. Given the Environment Agency's preference for passive control, alternative methods that can be used to improve the efficiency of the storage area, such as automated penstocks, have not been considered at this stage. The spillway crest selected after initial testing was set to have a length of 35m with crest level of 79.3m AOD, which would overtop in a 1.33% AEP but with a 4m long notch set to have a crest level of 78.2m that would just start to overtop in a 50% AEP;
- For the 1% AEP, it was found that even when replacement floodplain storage was used at the sludge ponds, the increase in downstream flood risk was not mitigated. This was found to be due to the lower bank levels of the diversion channel causing increased overtopping and increased flow out of the pond area, over an area of higher ground and onwards through the culvert under the Network Rail embankment and into the flood relief channel. In the 10% AEP in the baseline situation this same flow route, in combination with flow directly entering the upstream end of the flood relief channel from the Tame, results in a small area of flooding of land adjacent to a depot between the flood relief channel and the Tame. Given the diversion channel causes increased flow through the culvert and increased flow along the flood relief channel in the 1% AEP, the downstream water levels and flood extents increase. A similar effect was observed for the 5% AEP but with lesser effect given the overland flooding from the Tame downstream of the Network Rail culvert. For larger flood events, the flow through the culvert is slightly decreased compared to the baseline situation because the upstream floodplain is sufficiently filled to become controlled by the water level in the River Tame as the river banks become drowned out; and
- Downstream protection such as flood walls was not considered as water levels, and therefore residual flood risk, would still be raised above the baseline situation. Options to throttle the culvert were also discounted as this could increase upstream flood risk for larger flood events and could increase the risk of blockage. After some testing with the model, it was found that the increase in flood risk due to flow through the culvert in the 10% AEP could be mitigated by limited raising of ground levels along the high point between the pond and the culvert. This was represented in the model by raising ground levels to approximately 79.1m AOD along a 30m length (raised from their current level by a maximum of approximately 0.7m along a 10m section of this length).

11.1.7 The replacement floodplain storage option with the limited raising of ground levels downstream of the pond was simulated for return periods between 50% AEP and 0.1% AEP, each for 7.75, 10.75 and 15.75hr storm durations. The results show the replacement floodplain storage option would not increase upstream or downstream flood risk. For most return periods, a small reduction in flood risk would be achieved.

11.1.8 Further refinements of ground levels and the spillway geometry could enable a reduction in the extent of replacement floodplain storage at the disused treatment works.

11.1.9 Although this option has been developed, and has been shown that this solution can provide the necessary storage, the land is highly contaminated. The remediation costs of this are substantial, and due to the proximity of the works to the River Tame, there would be increased environmental risks during the remediation process.

Figure 13: Flow paths

11.1.10 The volume of storage required is significant, in the region of 400,000m^3 of highly contaminated material. This has a significant cost implication and also comes with a high risk to the environment due to the remedial works occurring directly adjacent to the River Tame. Therefore, this option is discounted, and the preferred option is option 3 of a straight alignment running in parallel to the Proposed Scheme.